COMMONWEALTH of VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION: MATERIALS

REPORT COVER SHEET

Geotechnical Engineering Report
Bridges B602, B603, B604, and B605
August 2015
AECOM Technical Services, Inc.
Virginia Beach, Virginia

COMMONWEALTH OF VIRGINIA
JAMES EARL SHERRING
Lic. No. 025367
PROFESSIONAL ENGINEER

James E Sherring
2016-06-03

AECOM
Hunt Valley, Maryland
Geotechnical Engineer

Responsible for Pages: All

Project Description:	**RTE 264 – Interchange Improvements – 64 WB Ramp to 264 EB**
From:	**0.757 Mile South of Curlew Drive**
To:	**0.832 Mile East of WBL I-64**
Project UPC No.:	**57048**

TABLE OF CONTENTS

APPENDICES

PRELIMINARY GEOTECHNICAL REPORT
I-264, PROJECT 0264-122-108
BRIDGES B602, B603, B-604 & B605
NORFOLK, VIRGINIA

INTRODUCTION

This report presents the results of our preliminary geotechnical design for proposed bridges associated with proposed improvements to the I-64 / I-264 interchange in the City of Norfolk, Virginia. These bridges will form integral parts of new ramps from westbound I-64 to eastbound I-264 and the proposed Collector-Distributor (CD) Road along I-264, as well as a new ramp from eastbound I-264 to the proposed CD Road. A general description of the subsurface conditions and geotechnical recommendations for the design of the foundations for these structures are included herein.

The borings and laboratory testing considered in our evaluations were completed by HDR Engineering, Inc. under their contract with VDOT. In additional to completing a comprehensive investigation program, HDR's scope included evaluation of the subsurface conditions with respect to embankments and roadway construction, with emphasis on settlement and global stability. The scope included in the current design effort is limited to the proposed structures, such as bridges, retaining walls, and larger drainage conveyances. HDR completed Geotechnical Engineering Reports (GERs) for the interchange improvement program, divided into seven Project Study Areas (PSAs). Of those, the following, all dated September 20, 2013, are pertinent to this contract:

GER No. 2: Ramps and Channel
GER No. 3: I-64, D7, and Bridge
GER No. 7: Curlew

This report is not final. Rather, it is intended to support our ongoing design, and provide VDOT with preliminary information regarding our approach to the design of the four bridges in this contract. A separate geotechnical report is being submitted which addresses the associated retaining walls.

PROPOSED CONSTRUCTION

The site lies near the eastern limits of the City of Norfolk; however, future contracts within the overall program extend eastward into the City of Virginia Beach. This project, located in the southeast quadrant of the I-64/I-264 interchange, includes four new bridges, as listed in the following table:

Table 1: Project 0264-122-108, Proposed Bridges

Bridge No.	Location	Baseline	Approx. Station Limits	Approx. Length (ft)	No. Spans
B602	I-64 WB, ramp to I-264 EB	Ramp D7	19+19 to 43+96	2,477	21
B603	Ramp D7 to I-264 CD Road EB*	Ramp D7 CD	16+27 to 27+17	1,090	12
B604	Ramp D7 over Kempsville Road	Ramp D7	47+63 to 49+60	197	2
B605	I-264 to I-264 CD Road EB	I-264 CD	26+10 to 42+22	1,612	14

*Note: Bridge B603 begins at B602 Pier 12, and ends at B605 Pier 4. It has no abutments.

The proposed bridges and associated roadway improvements are intended to improve traffic flow, increase capacity, and enhance safety for the traveling public. The existing elevations will generally provide sufficient grade separation for clearance under the bridges without the need for extensive grading; however, limited widening and/or approach fills are needed at the abutments of some the bridges.

Due to the poor soil conditions near the south end of bridge B602, that structure was extended further south to preclude the need for a high approach embankment. With this configuration, the bottom flanges of some of the beams were below present grades, especially approaching the abutment. To address this, grades in the area will be lowered to provide clearance below the beams and prevent contact with the ground, and to facilitate future inspection and maintenance. As this work will remove the eastern side slope of the I-64 embankment, a new retaining wall is required to maintain the integrity of the existing fill.

The anticipated design is for each bridge pier to be supported by three to six columns. The columns will generally be 4'-0" in diameter, with several more heavily-loaded piers upsized 6- to 12-inches larger. Each column will be supported by a single drilled shaft, sized at least 6 inches larger than its corresponding column. We have tentatively considered driven steel H-piles to support the abutments; however, drilled shafts are also feasible, and will be considered. Anticipated factored loads are discussed later in this report.

SITE CONDITIONS

The site generally consists of the rights of way for the existing highways and local roads; however, as the area is fully developed, some land acquisitions were needed to provide space for the new construction. The acquisitions and easements were secured during earlier planning and preliminary design stages of the project.

Surface features include the current D7 ramp and bridges over Curlew Drive and Kempsville Road; however, the proposed construction will run parallel and east/south of the existing facilities. These areas primarily consist of wooded land. A prominent site feature is Noseh's Creek, which enters the site where I-64 crosses Curlew Drive, parallels Ramp D7, then exits to the north where Kempsville Road crosses I-264.

Approximate surface elevations range from 8, to 40, with the higher elevations the result of fill placed for the existing approaches and abutments.

SITE GEOLOGY

The following paragraphs comprise a brief overview of the geologic setting at the site. Detailed discussions of the local geology are provided in the previously referenced Geotechnical Engineering Reports prepared by HDR.

The site lies in the Coastal Plain which encompasses the eastern portion of the state, beginning at the "Fall Line," roughly approximated by Interstate I-95, where the coastal sediments transition to bedrock-derived Piedmont materials.

Published geologic information covering the Virginia Coastal Plain indicates that the site is underlain by the geologically recent Tabb Formation, and the more ancient Yorktown Formation, a subunit of the Chesapeake Group, at depth.

The Quaternary age Tabb Formation is present to approximate elevation -100 to -110. Subunits of this formation includes the Lynnhaven and Sedgefield Members (Upper Pleistocene) encountered throughout all of the shallower borings and much of the deeper borings. These consist primarily of silty and clayey sands, which are very loose to loose nearer the surface, transitioning to medium dense nearer the base. Occurring alternately with the sands are very soft to medium stiff clays, generally exhibiting high plasticity.

Below these geologically recent strata lies the Tertiary age Yorktown Formation, which consists of gray silty sand interbedded with sandy silt. The Yorktown contains marine shell fragments, which are typical with most subunits of the Chesapeake Group. These strata are preconsolidated, and as a result are medium dense to dense in situ.

Although not noted in the bridge borings, it is anticipated that very recent Holocene age Alluvium is present, particularly along the present and former banks of the stream. Alluvial materials of this type consist of gravel, sand, silt, and clay of highly variable composition and sorting. Artificial fill placed to form the present highway embankments is also present along the right of way. Considering the age of the roadway, the old fill strata is expected to be highly variable, and may not meet current standards for compacted highway embankment fill.

SUBSURFACE EXPLORATION

As discussed above, VDOT engaged HDR Engineering, Inc. (HDR) to perform subsurface explorations, laboratory soil testing, and prepare geotechnical reports. HDR's scope was to use the subsurface and laboratory data to complete engineering analyses addressing roadways and embankments; specifically design considerations such as global stability and settlement. Their reports were also to present and transmit the data for use in the present design effort for major structures such as bridges and retaining walls. Since these voluminous documents have already been submitted and accepted by VDOT, they are not reproduced herein; however, they should be considered incorporated into this report by reference.

SPT Borings

The explorations considered for the bridges were generally limited to those drilled to the Yorktown bearing stratum, which begins in the vicinity of elevation -100. These subject borings were drilled to elevations ranging from -100 to about -130.

CPT Probes

A series of CPT (Cone Penetrometer Test) probes were performed to supplement the SPT borings These explorations, which included tip resistance, sleeve friction and selected pore pressure dissipation measurements, extended to approximate depths of 105 to 145 feet below the ground surface.

Laboratory Testing

The laboratory testing program, generally performed by S&ME or GET Solutions, included extensive index property tests (gradation analyses, Atterberg limits determinations (liquid and plastic), and natural moisture contents). The laboratory program also included compaction testing (moisture versus compacted density), California bearing ratio, chemical tests (pH, resistivity, sulfides, chlorides, organics), one-dimensional consolidation tests, and U-U triaxial tests.

The results of these tests are included in the Appendices of the HDR geotechnical reports

SUBSURFACE CONDITIONS

The conditions indicated by the borings are consistent with those described in the geologic discussion, and are discussed in considerable detail in the HDR Reports. Specifically, much of the near-surface zones consist of highly variable artificially placed fill. At the higher elevations, this probably represents embankment fill placed during earlier highway construction. At lower elevations, the fill could be the result of either earlier roadway construction, or the myriad development projects which have occurred over the years.

Below the fill, interbedded layers of clays and sands are present down to the Yorktown Formation. These layers can be divided in the upper sand and clay layers, characterized by very soft and loose consistencies, and the lower sand and clay layers, which are typically loose to medium dense, or soft to medium stiff, with sporadic stronger and weaker outliers. All of these natural strata represent the Quaternary geologic units described above.

The deepest borings penetrated the Yorktown Formation. These strata are mostly medium dense to dense silty or clayey sands, with sporadic shell fragments. Blow counts range between about 10 to nearly 50, and average in the mid 20's. The significant presence of fines coupled with the known preconsolidation stress history of these soils make these materials somewhat unusual, in that their strength is often greater than the SPT blow counts would suggest. Similarly, despite their typically sandy and shelly composition, they tend to be quite impermeable.

The subsurface data was generalized into distinct strata for preliminary design, with parameters assigned for each layer. The generalized stratification, and the soil properties assigned, are tabulated in the Appendix.

GEOTECHNICAL RECOMMENDATIONS

SITE PREPARATION AND GRADING

The bridge sites should be maintained in condition that will assure proper drainage and comply with applicable environmental regulations throughout construction. Based on the design using drilled shafts and H-piles, we expect that excavation will generally be limited to the approaches and abutments, and except where the grade is being lowered, are not expected to be deep.

To assure safety, all excavations must be shored or appropriately laid back to prevent sloughing or lateral displacement and to maintain safe working conditions. Site work must comply with OSHA regulations, specifically 29 CFR Part 1926, which requires that the soil be classified in the field by a "competent person." Because a competent person, as defined by the regulations, must have the authority to take prompt corrective action, personnel filling such a role should be employed or retained by the contractor and be on-site on a regular basis during performance of the work.

Design of temporary support measures are generally the responsibility of the construction contractor; the permanent top-down retaining wall is being designed by AECOM and is discussed in the geotechnical report addressing walls. Any required excavation support, both temporary and permanent, shall be designed in accordance with applicable AASHTO, VDOT and industry standards. The contractor-designed temporary support may be accomplished using sheet piles, timber sheeting, or soldier piles and lagging, or other feasible means. Sheeting may need to be braced or tied back, depending on construction sequence and the soil heights to be retained.

After clearing and grubbing the limited areas to receive new fill, the stripped surfaces should be examined by a geotechnical engineer to assure the exposed grade is suitable to begin placement of new fill. Any highly organic or debris-laden soils should be undercut to reasonably clean materials. It must be anticipated that even with removal of organic and other heterogeneous materials, the surface will be very moist to wet, and very weak. As such, establishing a base acceptable to begin filling will most likely require use of special procedures, such as end-dumping a "bridge lift" and displacing the muck or mud, the use of geotextile reinforcement, or perhaps other methods. Whichever method is used, the base of the embankment should consist of freely draining granular materials to provide strength, prevent wicking of moisture up through the fill, and to provide a conveyance for groundwater escaping the underlying soils as they are compressed.

Depending on availability, construction schedules, and prevailing weather conditions at the time of construction, it may prove beneficial to use alternative fill materials that are not susceptible to moisture, and can stabilize soft subgrades. Such materials include a number of aggregates mixtures, fly-ash-based flowable fill, and proprietary lightweight products (geo-foam, foamed concrete, etc.).

Standard earthwork procedures require fill placement to be controlled, and its degree of compaction tested, to assure minimum standards are met. Specifically, each lift of fill must be compacted to at least 95 percent of the theoretical maximum dry density, as determined by VTM-1 or VTM-12 (~standard Proctor), at a moisture within 20 percent of the optimum moisture content. The subgrade, as well as all base and subbase layers, shall be compacted to as required by VDOT's Road and Bridge Specifications. Materials shall be placed in approximately level layers not exceeding 8-inches, loose thickness, with the density and moisture after compaction verified by a qualified soils technician prior to placement of successive lifts.

BRIDGE FOUNDATIONS

Materials competent to support the anticipated loads are deep, generally starting at least 90 to 110 feet below grade, and as such deep foundations are required. In consideration of the subsurface conditions, we recommend that the bridge piers be founded on drilled shafts, and the abutments founded on driven H-piles, bearing in the medium dense to dense pre-consolidated Yorktown Formation stratum at these depths.

The factored structural loads are still being developed; however, the structural engineers have indicated that the per-shaft reactions for the piers are expected to range between approximately 500 and 1300 kips, with isolated cases loaded as heavily as 1600 kips. Preliminary abutment design is being based upon steel H-piles supporting factored loads on the order of 200 to 250 kips; however, pile size and spacing can be revised to accommodate different loads.

AECOM's preliminary design is based on support of the substructures with shafts or H-piles, as indicated above. Pre-stressed precast concrete (PPC) piles were also considered for the abutments; however, the great depths to the bearing stratum and the potential for developing significant negative skin friction loads (drag loads) in the upper strata override the economic benefits typically associated with this pile type. Constructability, splicing, and safety considerations also discourage use of PPC piles at this project.

Drilled Shafts

The preliminary design is for each pier structure to be supported by a row of three to six columns. Each column will be supported by a single drilled shaft. The minimum drilled shaft size of 4'-6" was selected to be larger than the columns being supported for constructability concerns, as well as to develop the axial, shear and moment capacities required by design under the LRFD code.

Evaluation of factored resistances for axial loading was performed in general conformance with "LRFD Bridges Design Specifications, 2014," including interim revisions, published by AASHTO. The code endorses use of the 2010 FHWA Publication covering the design and construction of drilled shafts, by Brown, Turner and Castelli;[1] however, the resistance factors used in our analysis are those specified in the current (2014) Code. Lateral loads and moment

[1] Brown, Dan A., John P. Turner and Raymond J. Castelli, Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA NHI-10-016, 2010.

capacity analysis were performed as a component of the structural analyses using the FB-MultiPier software application.

The code requires the following resistance factors when calculating the geotechnical axial capacity of a drilled shaft, as listed in Table 10.8.3.6.3-1:

Skin Resistance, Cohesive Soil:	0.45
Skin Resistance, Non-Cohesive Soil:	0.55
Tip Resistance, Cohesive Soil:	0.40
Tip Resistance, Non-Cohesive Soil:	0.50

Our analyses indicate that the anticipated loads can be supported by shafts in the 4.5- to 5.0-foot diameter range. To assure proper bearing, we recommend that the shaft tips extend at least 5 feet into the Yorktown Formation. The more highly loaded shafts may need to be extended deeper into the Yorktown soils to develop skin friction therein, or may need to be moderately belled (up to 3 feet larger than shaft diameter) to develop the necessary factored resistance. Spreadsheets showing example calculations are provided in the Appendix.

Hydrostatic pressures in shallower holes can be overcome by the use of casings and/or taking advantage of relatively impermeable materials; however, the depths anticipated on this project will result in very high pressures, which creates a significant risk of side collapse, blow-in, or boils. Accordingly, we recommend that the "wet method," in which a bentonite or polymer slurry is used to maintain the holes during drilling, reinforcement placement and concreting, be mandated.

All shaft excavations should be maintained by slurry during construction. Even so, it is essential that reinforcing steel and concrete be placed as soon as practicable after drilling to prevent degradation of the shaft excavations. We anticipate that, as is typical with this method, a short length of casing is placed at the top of the excavation after commencing drilling to maintain the size and shape of the hole, with slurry used throughout the remainder of the shaft. Slurry is pumped from a nearby tank or reservoir as the hole progresses, which is recovered for reuse during the concreting process.

With the sensitive subsoils, and the loss of integrity and capacity that can occur during foundation construction, the specifications should require the contractor to have demonstrated experience installing deep drilled shafts in the Hampton Roads area. Additionally, the on-site inspection personnel should be experienced in identifying competent Yorktown Formation soils, which will have to be identified from the spoils. Careless installation procedures and inexperienced personnel could result in soil inclusions and shafts bearing on disturbed materials, as well as other defects.

There may be instances where more extensive casing depth, either temporary or permanent, may be warranted to protect nearby utilities, existing structures, etc. In such cases, the casing is extended to greater depths and/or left in place, to minimize the potential for lateral or axial loading of the exiting construction. The cased zone should be evaluated on a case by case basis, as necessary.

Abutment Foundations – Driven Piles

The abutments will be supported by both plumb piles and battered piles. Typical structural design uses the battered piles to resist lateral and transverse loading; all piles will provide varying degrees of axial support. We expect lateral and transverse reactions per pile to be inconsequential from a geotechnical standpoint; however, we will evaluate such loading, if applicable, prior to finalizing the design.

The estimated maximum factored load of 250 kips provided by the structural engineer was used to calculate the required nominal pile resistance. Applying a resistance factor (Φ) of 0.65, per the LRFD Code, provides a per-pile nominal capacity of 385 kips per pile.

The use of Φ=0.65 as the resistance factor, in accordance with AASHTO LRFD Table 10.5.5.2.3-1, requires dynamic testing during test pile installation, coupled with signal matching of the data to verify pile capacity. As noted in the AASHTO table, dynamic testing and signal matching must be performed on two piles for every site condition, and for at least 2 percent of the total production piles, whichever is greater. We would envision dynamic testing of two (2) to three (3) piles from each abutment substructure, which meets or exceeds the minimum requirements.

Nominal resistance values were evaluated to determine predicted pile penetration depths using the “APILE” computer program developed by Ensoft. Inc. The resulting estimated pile embedment to obtain the required nominal resistance of 385 kips, is 132 feet at Abutment A of bridge B605, based upon a 12-inch HP12x74 pile section. The surface elevations vary; our preliminary model used a surface elevation of 18, and neglected skin friction capacity in the upper 50 feet. The embedment results in an estimated pile tip elevation of -114. APILE output sheets showing the example calculations are provided in the Appendix.

The generally weak soil conditions throughout much of the profile do not suggest unusual or difficult driving; however, should there be any unusual observations during pile installation, such as unexpected soft or hard zones, lateral drift or “kick,” etc., they must be evaluated by the geotechnical engineer to determine if corrective procedures or additional evaluations are warranted.

The estimated tip elevations indicate the depths at which the factored resistance values are predicted to develop for the recommended pile sizes. Pile order lengths should be established by pre-production test/indicator piles; the test piles in turn should be ordered somewhat longer than the estimated penetration length, typically at least 10 feet, but perhaps longer as determined by the contractor's wave equation analyses, to ensure sufficient length to reach capacity during initial driving.

The AASHTO code allows use of higher resistance factors for higher quality control levels of production piles. A factor of Φ=0.75 may be used if dynamic testing is performed on all production piles, or if one successful static load test is completed for each site condition. A resistance factor of Φ=0.80 may be used if both a successful static load test, and at least (2) dynamic tests, are completed for each site condition. While static load tests are often not cost effective, in some cases the additional dynamic testing to allow a factor of Φ=0.75 may be.

The pier pile capacities do not consider any downdrag (also called negative skin friction or drag load), but they do neglect skin friction capacity in the upper 50 feet of pile embedment. Depending on construction scheduling, the approach fills near the abutments may potentially create a drag load due to compression of the weak upper strata. If site grading, structural configuration, etc. requires considering downdrag, appropriate skin fraction values can be determined at that time. We expect that coating the upper pile sections with a proprietary product designed to reduce skin friction will be cost effective; however, this requires further study once quantities and costs can be more accurately estimated. Other possible methods include ensuring all settlement has occurred prior to driving, or to complete initial drives, construct the fill, and then restrike the piles after settlement is complete to "break" the drag load.

As a precursor to test pile installation, the contractor shall propose equipment for driving, substantiated by wave equation analysis. During driving and re-striking the test piles, the entire procedure shall be monitored by dynamic testing, to verify pile stresses and capacities, as well as hammer efficiency and performance. After the minimum wait time and re-striking, the nominal pile capacity shall be refined by signal matching analysis of the dynamic data. Upon review of the driving records, dynamic test data, and signal matching analyses, the dynamic testing engineer shall propose the criteria for installing production piles, subject to review and approval by VDOT and the geotechnical engineer of record.

APPENDIX A

Geotechnical Engineering Reports by HDR (Separate Documents)

APPENDIX B

Soil Parameter Summary Tables

SOIL PROPERTIES FOR FB-PIER PILE GROUP ANALYSIS

BRIDGE B602

Stratum		Elevation Range			Total Unit Weight	Bouyed Unit Weight	Angle of Internal Friction	Undrained Shear	Poisson's Ratio	Lateral Subgrade Modulus	Young's Modulus	Shear Modulus	Strain Parameters	
No.	Description	Top	Bottom	Type	γ (pcf)	γ' (pcf)	ϕ' (deg)	S_u (psf)	ν	k (pci)	E (ksf)	G (psi)	ε_{50}	ε_{100}
1	Fill	(varies)	0.0	Fill	115	52.6	28	-	0.27	60	150	250	-	-
2	Norfolk/Tabb	0.0	-15.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
3	Norfolk/Tabb	-15.0	-25.0	Clay	100	37.6	-	144	0.35	50	58	148	0.03	0.09
4	Norfolk/Tabb	-25.0	-45.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
5	Norfolk/Tabb	-45.0	-95.0	Clay	100	37.6	-	144	0.35	50	58	148	0.03	0.09
6	Yorktown Fm	-95.0	<-95	Sand	118	55.6	32	-	0.35	75	250	643	-	-

BRIDGE B603

Stratum		Elevation Range			Total Unit Weight	Bouyed Unit Weight	Angle of Internal Friction	Undrained Shear	Poisson's Ratio	Lateral Subgrade Modulus	Young's Modulus	Shear Modulus	Strain Parameters	
No.	Description	Top	Bottom	Type	γ (pcf)	γ' (pcf)	ϕ' (deg)	S_u (psf)	ν	k (pci)	E (ksf)	G (psi)	ε_{50}	ε_{100}
1	Fill	(varies)	0.0	Fill	115	52.6	28	-	0.27	60	150	250	-	-
2	Norfolk/Tabb	0.0	-20.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
3	Norfolk/Tabb	-20.0	-30.0	Clay	105	42.6	-	216	0.37	150	-	365	0.02	0.06
4	Norfolk/Tabb	-30.0	-50.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
5	Norfolk/Tabb	-50.0	-100.0	Clay	105	42.6	-	216	0.27	150	-	365	0.02	0.06
6	Yorktown Fm	-100.0	<-100	Sand	118	55.6	32	-	0.35	75	250	643	-	-

SOIL PROPERTIES FOR FB-PIER PILE GROUP ANALYSIS

BRIDGE B604

Stratum		Elevation Range			Total Unit Weight	Bouyed Unit Weight	Angle of Internal Friction	Undrained Shear	Poisson's Ratio	Lateral Subgrade Modulus	Young's Modulus	Shear Modulus	Strain Parameters	
No.	Description	Top	Bottom	Type	γ (pcf)	γ' (pcf)	ϕ' (deg)	S_u (psf)	ν	k (pci)	E (ksf)	G (psi)	ε_{50}	ε_{100}
1	Fill	(varies)	0.0	Fill	115	52.6	28	-	0.27	60	150	250	-	-
2	Norfolk/Tabb	0.0	-16.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
3	Norfolk/Tabb	-16.0	-20.0	Clay	100	37.6	-	144	0.35	50	58	148	0.03	0.09
4	Norfolk/Tabb	-20.0	-60.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
5	Norfolk/Tabb	-60.0	-105.0	Clay	100	37.6	-	144	0.35	50	58	148	0.03	0.09
6	Yorktown Fm	-105.0	<-105	Sand	118	55.6	32	-	0.35	75	250	643	-	-

BRIDGE B605

Stratum		Elevation Range			Total Unit Weight	Bouyed Unit Weight	Angle of Internal Friction	Undrained Shear	Poisson's Ratio	Lateral Subgrade Modulus	Young's Modulus	Shear Modulus	Strain Parameters	
No.	Description	Top	Bottom	Type	γ (pcf)	γ' (pcf)	ϕ' (deg)	S_u (psf)	ν	k (pci)	E (ksf)	G (psi)	ε_{50}	ε_{100}
1	Fill	(varies)	0.0	Fill	115	52.6	28	-	0.27	60	150	250	-	-
2	Norfolk/Tabb	0.0	-15.0	Sand	105	42.6	29	-	0.25	30	100	278	-	-
3	Norfolk/Tabb	-15.0	-20.0	Clay	100	37.6	-	144	0.35	50	58	148	0.03	0.09
4	Norfolk/Tabb	-20.0	-50.0	Sand	110	47.6	29	-	0.30	40	150	400	-	-
5	Norfolk/Tabb	-50.0	-105.0	Clay	105	42.6	-	144	0.35	50	58	148	0.03	0.09
6	Yorktown Fm	-105.0	<-105	Sand	118	55.6	32	-	0.35	75	250	643	-	-

APPENDIX C

Shaft Capacity Analysis (Spreadsheets)

Drilled Shaft Capacity Evaluation - I-264 Project **B602**

Location		B602; DS-1	
Surface Elev.		10	ft
Shaft Diameter	D	4.5	ft
Atmospheric Pressure	pa	2.12	ksf

ER, in % of theoretical energy

Water Elevation: 0

Stratum No	Mat'l Type	Top Elev	Bttm Elev	Thick-ness	Depth of Mid-point	Total unit wt (pcf)	Eff Unit wt (pcf)	Su (ksf)	Su/pa (ksf)	Alfa factor, α (dim)	$N1_{60} = C_N * N_{60}$ SPT N value corrected for overburden and hammer $(N_1)_{60}$	N60 = (ER/60%) N SPT N value corrected for hammer N_{60}	0.6 for clean sands; 0.8 for SM and sandy ML m	=27.5 + 9.2 log $N1_{60}$ Calculated Friction angle of soil (degrees) ϕ'_f / σ'_f	consolidation stress =pa*0.47*N60^m (sands) σ'_p (ksf)	Effective stress at mid-layer σ'_v (ksf)	Beta Factor β (dim)	Side resist. q_s (ksf)	Nominal side resistance R_s (kips)	Resistance Factor (φ)	Factored Side Resistance R_s (kips)	Cumulative Factored Side Resistance (kips)	
1	Cohesionless	20	0	20	10	115	115				1	1	0.8	27.50	0.996	1.15	0.262	0.302	85.27	0.55	46.90	0.0	Neglect
2	Cohesionless	0	-18	18	-9	105	42.6				2	2	0.8	30.27	1.735	2.69	0.232	0.624	158.83	0.55	87.36	87.4	
3	Cohesive	-18	-25	7	-21.5	100	37.6	0.144	0.0679	0.5500						3.2		0.079	7.84	0.45	3.53	90.9	
4	Cohesionless	-25	-50	25	-37.5	105	42.6				2	2	0.8	30.27	1.735	3.87	0.193	0.748	264.20	0.55	145.31	236.2	
5	Cohesive	-50	-75	25	-62.5	100	37.6	0.144	0.0679	0.5500						4.87		0.079	27.99	0.45	12.60	248.8	
6	Cohesive	-75	-105	30	-90	100	37.6	0.144	0.0679	0.5500						5.72		0.079	33.59	0.45	15.12	263.9	
7	Cohesionless	-105	-110	5	-107.5	118	55.6				17	28	0.8	38.82	14.327	6	0.518	3.108	219.73	0.55	120.85	384.8	
8	Cohesionless	-110	-135	25	-122.5	118	55.6				19	32	0.8	39.26	15.942	7.5	0.484	3.627	1281.98	0.55	705.09	1,089.8	
9																							
10																							
11																							
12																							

Tip - Cohesionless

Belled? No   Bell Diameter

Z= 115 ft, below ground surface

25 N60 (Enter N60 for zone 2 diameters below tip elev.)

q_p= 30 =1.2 * N_{60} (ksf) N corrected for hammer only

R_p= 477 kips

φ = 0.5

R_R = 239 kips

Factored Side Resistance: 385 kips

Factored Tip Resistance: 239 kips

Total Factored Resistance **624** kips

RESISTANCE FACTORS: (Table 10.5.5.2.4-1)

Side - Clay	0.45
Side - Sand	0.55
Tip - Clay	0.4
Tip - Sand	0.5

N/A

Tip - Cohesive

Belled? No   5 Bell Diameter

Z= 100 ft, below ground surface

N_c= 9.00 = 6 * [1 + 0.2 (Z/D)] ≤ 9

-90.0   -99.0   Depth range of S_u values = 2 diameters below tip of shaft

S_u = 2 (from above, as applicable to depth - ksf). If Su < 0.50 ksf, use 0.67 * Su

q_p= 18 =N_c * S_u ≤ 80.0 ksf

R_p= 286 kips

φ = 0.40

C_N= 0.77 log (40/σ'_v) <2.0   (stress in ksf)

σ'_v = 6.5 ksf

C_N = 0.61 (dim)

N60: N = 24   32.0 = N60 (for auto hammer: 80%)

19 = $N1_{60}$

Drilled Shaft Capacity Evaluation - I-264 Project B602

Location		B602; DS-1	
Surface Elev.		10	ft
Shaft Diameter	D	5.5	ft
Atmospheric Pressure	pa	2.12	ksf

Water Elevation: 0

ER, in % of theoretical energy

Stratum No	Mat'l Type	Top Elev	Bttm Elev	Thick-ness	Depth of Mid-point	Total unit wt (pcf)	Eff Unit wt (pcf)	Su (ksf)	Su/pa (ksf)	Alfa factor, α (dim)	$N1_{60} = C_N * N_{60}$ SPT N value corrected for overburden and hammer $(N_1)_{60}$	N60 = (ER/60%) N SPT N value corrected for hammer N_{60}	0.6 for clean sands; 0.8 for SM and sandy ML m	=27.5 + 9.2 log $N1_{60}$ Calculated Friction angle of soil (degrees) ϕ'_f / σ'_f	consolidation stress =pa*0.47*N60^m (sands) σ'_p (ksf)	Effective stress at mid-layer σ'_v (ksf)	Beta Factor β (dim)	Side resist. q_s (ksf)	Nominal side resistance R_s (kips)	Resistance Factor (φ)	Factored Side Resistance R_s (kips)	Cumulative Factored Side Resistance (kips)	
1	Cohesionless	20	0	20	10	115	115				1	1	0.8	27.50	0.996	1.15	0.262	0.302	104.22	0.55	57.32	0.0	Neglect
2	Cohesionless	0	-18	18	-9	105	42.6				2	2	0.8	30.27	1.735	2.69	0.232	0.624	194.13	0.55	106.77	106.8	
3	Cohesive	-18	-25	7	-21.5	100	37.6	0.144	0.0679	0.5500						3.2		0.079	9.58	0.45	4.31	111.1	
4	Cohesionless	-25	-50	25	-37.5	105	42.6				2	2	0.8	30.27	1.735	3.87	0.193	0.748	322.91	0.55	177.60	288.7	
5	Cohesive	-50	-75	25	-62.5	100	37.6	0.144	0.0679	0.5500						4.87		0.079	34.21	0.45	15.40	304.1	
6	Cohesive	-75	-105	30	-90	100	37.6	0.144	0.0679	0.5500						5.72		0.079	41.05	0.45	18.47	322.6	
7	Cohesionless	-105	-120	15	-112.5	118	55.6				17	28	0.8	38.82	14.327	6	0.518	3.108	805.66	0.55	443.11	765.7	
8	Cohesionless	-120	-125	5	-122.5	118	55.6				19	32	0.8	39.26	15.942	7.5	0.484	3.627	313.37	0.55	172.36	938.0	
9																							
10																							
11																							
12																							

Tip - Cohesionless

Belled? Yes 7.5 Bell Diameter

Z= 115 ft, below ground surface

29 N60 (Enter N60 for zone 2 diameters below tip elev.)

q_p= 34.8 =1.2 * N_{60} (ksf) N corrected for hammer only

R_p= 1,537 kips

φ = 0.5

R_R = 769 kips

N/A

Tip - Cohesive

Belled? No 5 Bell Diameter

Z= 100 ft, below ground surface

N_c= 9.00 = 6 * [1 + 0.2 (Z/D)] ≤ 9

-90.0 -101.0 Depth range of S_u values = 2 diameters below tip of shaft

S_u = 2 (from above, as applicable to depth - ksf). If Su < 0.50 ksf, use 0.67 * Su

q_p= 18 =N_c * S_u ≤ 80.0 ksf

R_p= 428 kips

φ = 0.40

Factored Side Resistance:	938	kips
Factored Tip Resistance:	770	kips
Total Factored Resistance	**1708**	kips

C_N= 0.77 log (40/σ'_v) <2.0 (stress in ksf)

σ'_v = 6.5 ksf

C_N = 0.61 (dim)

N60: N = 24 32.0 = N60 (for auto hammer: 80%)

19 = $N1_{60}$

RESISTANCE FACTORS: (Table 10.5.5.2.4-1)

Side - Clay	0.45
Side - Sand	0.55
Tip - Clay	0.4
Tip - Sand	0.5

APPENDIX D

APILE Output
(Abutment Piles)

```
=========================================================================
                APILE for Windows, Version 2014.6.10

                 Serial Number :  136155605

                 A Program for Analyzing the Axial Capacity
                 and Short-term Settlement of Driven Piles
                           under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2014
                     All Rights Reserved

=========================================================================


This program is licensed to :

AECOM
Hunt Valley, MD

Path to file locations      : C:\Users\James_Sherring\Documents\
Name of input data file     : B605 A.ap6d
Name of output file         : B605 A.ap6o
Name of plot output file    : B605 A.ap6p

-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  August 31, 2015     Time:  14:33:55


1
       ********************
       * INPUT INFORMATION *
       ********************



       I64/264 - Abutment A

       DESIGNER   : JES

       JOB NUMBER :


       METHOD FOR UNIT LOAD TRANSFERS :

       - FHWA (Federal Highway Administration)
         Unfactored Unit Side Friction and Unit Side Resistance are used.


       COMPUTATION METHOD(S) FOR PILE CAPACITY :

       - FHWA (Federal Highway Administration)


       TYPE OF LOADING :
```

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 144.00 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 140.00 FT.
- PILE STICKUP LENGTH, PSL = 2.00 FT.
- ZERO FRICTION LENGTH, ZFL = 50.00 FT.
- PERIMETER OF PILE = 48.00 IN.
- TIP AREA OF PILE = 144.00 IN2
- INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	115.00	28.00	0.00
18.00	SAND	0.00	115.00	28.00	0.00
18.00	SAND	0.00	42.60	28.00	0.00
33.00	SAND	0.00	42.60	28.00	0.00
33.00	CLAY	0.00	37.60	0.00	0.00
38.00	CLAY	0.00	37.60	0.00	0.00
38.00	SAND	0.00	47.60	29.00	0.00
68.00	SAND	0.00	47.60	29.00	0.00
68.00	CLAY	0.00	42.60	0.00	0.00
123.00	CLAY	0.00	42.60	0.00	0.00
123.00	SAND	0.00	55.60	32.00	0.00
150.00	SAND	0.00	55.60	32.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.14	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.14	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.14	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.14	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
18.00	1.000	1.000
18.00	1.000	1.000
33.00	1.000	1.000
33.00	1.000	1.000
38.00	1.000	1.000
38.00	1.000	1.000
68.00	1.000	1.000
68.00	1.000	1.000
123.00	1.000	1.000
123.00	1.000	1.000
150.00	1.000	1.000

1

****
* COMPUTATION RESULT *
****

****
* FED. HWY. METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.7	0.7
1.00	0.0	1.4	1.4
2.00	0.0	2.8	2.8
3.00	0.0	4.2	4.2
4.00	0.0	5.6	5.6
5.00	0.0	7.0	7.0
6.00	0.0	8.4	8.4
7.00	0.0	9.8	9.8
8.00	0.0	11.2	11.2
9.00	0.0	12.2	12.2
10.00	0.0	12.9	12.9
11.00	0.0	13.2	13.2
12.00	0.0	13.3	13.3
13.00	0.0	13.3	13.3
14.00	0.0	13.3	13.3

15.00	0.0	13.3	13.3
16.00	0.0	13.3	13.3
17.00	0.0	13.3	13.3
18.00	0.0	13.3	13.3
19.00	0.0	13.3	13.3
20.00	0.0	13.3	13.3
21.00	0.0	13.3	13.3
22.00	0.0	13.3	13.3
23.00	0.0	13.3	13.3
24.00	0.0	13.3	13.3
25.00	0.0	13.3	13.3
26.00	0.0	13.3	13.3
27.00	0.0	13.3	13.3
28.00	0.0	13.3	13.3
29.00	0.0	13.3	13.3
30.00	0.0	13.3	13.3
31.00	0.0	13.3	13.3
32.00	0.0	10.5	10.5
33.00	0.0	7.3	7.3
34.00	0.0	4.2	4.2
35.00	0.0	1.3	1.3
36.00	0.0	1.3	1.3
37.00	0.0	4.2	4.2
38.00	0.0	7.3	7.3
39.00	0.0	10.5	10.5
40.00	0.0	13.3	13.3
41.00	0.0	13.3	13.3
42.00	0.0	13.3	13.3
43.00	0.0	13.3	13.3
44.00	0.0	13.3	13.3
45.00	0.0	13.3	13.3
46.00	0.0	13.3	13.3
47.00	0.0	13.3	13.3
48.00	0.0	13.3	13.3
49.00	0.0	13.3	13.3
50.00	3.6	13.3	16.9
51.00	10.8	13.3	24.1
52.00	18.1	13.3	31.4
53.00	25.5	13.3	38.8
54.00	33.0	13.3	46.3
55.00	40.6	13.3	53.9
56.00	48.3	13.3	61.6
57.00	56.1	13.3	69.4
58.00	64.0	13.3	77.3
59.00	72.0	13.3	85.3
60.00	80.0	13.3	93.4
61.00	88.2	13.3	101.5
62.00	96.5	13.3	109.8
63.00	104.9	13.3	118.2
64.00	113.4	13.3	126.7
65.00	122.0	13.3	135.3
66.00	130.6	13.3	144.0
67.00	139.4	10.5	149.9
68.00	148.3	7.3	155.6
69.00	153.0	4.2	157.2
70.00	153.6	1.3	154.9
71.00	154.2	1.3	155.5
72.00	154.8	1.3	156.1
73.00	155.3	1.3	156.6
74.00	155.9	1.3	157.2

75.00	156.5	1.3	157.8
76.00	157.1	1.3	158.4
77.00	157.6	1.3	158.9
78.00	158.2	1.3	159.5
79.00	158.8	1.3	160.1
80.00	159.4	1.3	160.7
81.00	159.9	1.3	161.2
82.00	160.5	1.3	161.8
83.00	161.1	1.3	162.4
84.00	161.7	1.3	163.0
85.00	162.2	1.3	163.5
86.00	162.8	1.3	164.1
87.00	163.4	1.3	164.7
88.00	164.0	1.3	165.3
89.00	164.6	1.3	165.8
90.00	165.1	1.3	166.4
91.00	165.7	1.3	167.0
92.00	166.3	1.3	167.6
93.00	166.9	1.3	168.2
94.00	167.4	1.3	168.7
95.00	168.0	1.3	169.3
96.00	168.6	1.3	169.9
97.00	169.2	1.3	170.5
98.00	169.7	1.3	171.0
99.00	170.3	1.3	171.6
100.00	170.9	1.3	172.2
101.00	171.5	1.3	172.8
102.00	172.0	1.3	173.3
103.00	172.6	1.3	173.9
104.00	173.2	1.3	174.5
105.00	173.8	1.3	175.1
106.00	174.3	1.3	175.6
107.00	174.9	1.3	176.2
108.00	175.5	1.3	176.8
109.00	176.1	1.3	177.4
110.00	176.6	1.3	177.9
111.00	177.2	1.3	178.5
112.00	177.8	1.3	179.1
113.00	178.4	1.3	179.7
114.00	179.0	1.3	180.2
115.00	179.5	1.3	180.8
116.00	180.1	1.3	181.4
117.00	180.7	1.3	182.0
118.00	181.3	1.3	182.6
119.00	181.8	1.3	183.1
120.00	182.4	1.3	183.7
121.00	183.0	1.3	184.3
122.00	183.6	8.8	192.4
123.00	184.1	17.1	201.3
124.00	194.1	25.4	219.5
125.00	213.5	33.0	246.5
126.00	233.0	33.0	266.0
127.00	252.8	33.0	285.8
128.00	272.6	33.0	305.6
129.00	292.7	33.0	325.7
130.00	312.9	33.0	345.9
131.00	333.2	33.0	366.2
132.00	353.7	33.0	386.7
133.00	374.4	33.0	407.4
134.00	395.3	33.0	428.3

135.00	416.3	33.0	449.3
136.00	437.4	33.0	470.4
137.00	458.7	33.0	491.7
138.00	480.2	33.0	513.2

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

```
*************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
*************************************************
```

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
2	10	0.9025E+01		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
3	10	0.1796E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
4	10	0.1800E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01

			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
5	10	0.2553E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
6	10	0.3296E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
7	10	0.3300E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2445E-01
			0.0000E+00	0.4736E-01
			0.0000E+00	0.8709E-01
			0.0000E+00	0.1222E+00
			0.0000E+00	0.1528E+00
			0.0000E+00	0.3056E+00
			0.0000E+00	0.4584E+00
			0.0000E+00	0.7639E+00
			0.0000E+00	0.3056E+01
8	10	0.3553E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2445E-01
			0.0000E+00	0.4736E-01
			0.0000E+00	0.8709E-01
			0.0000E+00	0.1222E+00
			0.0000E+00	0.1528E+00
			0.0000E+00	0.3056E+00
			0.0000E+00	0.4584E+00
			0.0000E+00	0.7639E+00
			0.0000E+00	0.3056E+01
9	10	0.3796E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2445E-01
			0.0000E+00	0.4736E-01
			0.0000E+00	0.8709E-01
			0.0000E+00	0.1222E+00
			0.0000E+00	0.1528E+00
			0.0000E+00	0.3056E+00
			0.0000E+00	0.4584E+00
			0.0000E+00	0.7639E+00

			0.0000E+00	0.3056E+01
10	10	0.3800E+02		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
11	10	0.5303E+02		
			0.0000E+00	0.0000E+00
			0.1310E+01	0.1000E-01
			0.2621E+01	0.2000E-01
			0.5242E+01	0.4000E-01
			0.7863E+01	0.6000E-01
			0.1048E+02	0.8000E-01
			0.1179E+02	0.9000E-01
			0.1310E+02	0.1000E+00
			0.1310E+02	0.5000E+00
			0.1310E+02	0.2000E+01
12	10	0.6796E+02		
			0.0000E+00	0.0000E+00
			0.1183E+01	0.1000E-01
			0.2365E+01	0.2000E-01
			0.4730E+01	0.4000E-01
			0.7096E+01	0.6000E-01
			0.9461E+01	0.8000E-01
			0.1064E+02	0.9000E-01
			0.1183E+02	0.1000E+00
			0.1183E+02	0.5000E+00
			0.1183E+02	0.2000E+01
13	10	0.6800E+02		
			0.0000E+00	0.0000E+00
			0.1387E+01	0.2445E-01
			0.2311E+01	0.4736E-01
			0.3467E+01	0.8709E-01
			0.4161E+01	0.1222E+00
			0.4623E+01	0.1528E+00
			0.4161E+01	0.3056E+00
			0.4161E+01	0.4584E+00
			0.4161E+01	0.7639E+00
			0.4161E+01	0.3056E+01
14	10	0.9553E+02		
			0.0000E+00	0.0000E+00
			0.3000E+00	0.2445E-01
			0.5000E+00	0.4736E-01
			0.7500E+00	0.8709E-01
			0.9000E+00	0.1222E+00
			0.1000E+01	0.1528E+00
			0.9000E+00	0.3056E+00
			0.9000E+00	0.4584E+00
			0.9000E+00	0.7639E+00
			0.9000E+00	0.3056E+01
15	10	0.1230E+03		
			0.0000E+00	0.0000E+00
			0.2740E+01	0.2445E-01
			0.4567E+01	0.4736E-01

			0.6851E+01	0.8709E-01
			0.8221E+01	0.1222E+00
			0.9135E+01	0.1528E+00
			0.8221E+01	0.3056E+00
			0.8221E+01	0.4584E+00
			0.8221E+01	0.7639E+00
			0.8221E+01	0.3056E+01
16	10	0.1230E+03		
			0.0000E+00	0.0000E+00
			0.2547E+01	0.1000E-01
			0.5095E+01	0.2000E-01
			0.1019E+02	0.4000E-01
			0.1528E+02	0.6000E-01
			0.2038E+02	0.8000E-01
			0.2293E+02	0.9000E-01
			0.2547E+02	0.1000E+00
			0.2547E+02	0.5000E+00
			0.2547E+02	0.2000E+01
17	10	0.1365E+03		
			0.0000E+00	0.0000E+00
			0.3714E+01	0.1000E-01
			0.7429E+01	0.2000E-01
			0.1486E+02	0.4000E-01
			0.2229E+02	0.6000E-01
			0.2972E+02	0.8000E-01
			0.3343E+02	0.9000E-01
			0.3714E+02	0.1000E+00
			0.3714E+02	0.5000E+00
			0.3714E+02	0.2000E+01
18	10	0.1500E+03		
			0.0000E+00	0.0000E+00
			0.3728E+01	0.1000E-01
			0.7457E+01	0.2000E-01
			0.1491E+02	0.4000E-01
			0.2237E+02	0.6000E-01
			0.2983E+02	0.8000E-01
			0.3355E+02	0.9000E-01
			0.3728E+02	0.1000E+00
			0.3728E+02	0.5000E+00
			0.3728E+02	0.2000E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.2062E+01	0.7639E-02
0.4125E+01	0.1528E-01
0.8250E+01	0.3056E-01
0.1650E+02	0.1986E+00
0.2475E+02	0.6417E+00
0.2970E+02	0.1115E+01
0.3300E+02	0.1528E+01
0.3300E+02	0.2292E+01
0.3300E+02	0.3056E+01

LOAD VERSUS SETTLEMENT CURVE

****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.8272E+00	0.3289E-03	0.2700E-01	0.1000E-03
0.8272E+01	0.3289E-02	0.2700E+00	0.1000E-02
0.4136E+02	0.1645E-01	0.1350E+01	0.5000E-02
0.8279E+02	0.3290E-01	0.2700E+01	0.1000E-01
0.3779E+03	0.1548E+00	0.9204E+01	0.5000E-01
0.5643E+03	0.2706E+00	0.1166E+02	0.1000E+00
0.5731E+03	0.6747E+00	0.2211E+02	0.5000E+00
0.5795E+03	0.1177E+01	0.2849E+02	0.1000E+01
0.5840E+03	0.2179E+01	0.3300E+02	0.2000E+01